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NUMERICAL INVESTIGATIONS ON DYNAMIC BEHAVIOUR OF LUMPED MASS MODEL OF NON-STRUCTURAL ELEMENTS & 3D RC BUILDINGS

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Abstract

This study considers simple acceleration sensitive Non-structural elements (NSEs) with fundamental period $T_S \leq 0.07$seconds, modelled as a rigid cantilever with lumped mass at the top (stick model). Numerical modelling of building and NSE together (complete model) and nonlinear time history analyses using 11 natural ground motions are performed in commercial software PERFORM 3D. Parametric studies of NSEs with lumped mass of 1-10 percent total seismic weight of building (in increments of 1 percent) for each fundamental period of NSE (in increments of 0.01s) is carried out, using spectrally scaled ground motions. Variations in: (a) lateral force demand on NSE, (b) Component Amplification Factor (CAF), and (c) Floor Amplification Factor (FAF), are monitored. Results are compared with the values of above parameters estimated as per EAK (2000), Eurocode 8 (1998 (1):2004), ASCE 07 (2016), NZS 4219 (2009), and NZS 1170 (5) (2004). It is observed that FAF varies nonlinearly across the floors (and significantly in the lower floors), as against linear variation stipulated in the above codes. The average CAF for the ground motions considered ranged between 1 and 2 up to $T_s \leq$ 0.06s. However, it increased significantly for T_s = 0.07s, thereby complementing ASCE 07 stipulation of 0.06s limit for rigid NSEs. The value of 2 is consistent with NZS 1170(5):2004 and 1 with ASCE 07 and EAK (2000). Further, as the NSE weight increased, the fundamental period of the building increased, and the average CAF marginally decreased. And, seismic force demand imposed on all NSEs during nonlinear time history analysis exceeded code recommended seismic design force under most ground motions.

Keywords: Non-structural elements, stick model, design lateral force, Component Amplification Factor, Floor Amplification Factor

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1 INTRODUCTION

Non-structural elements (*NSEs*) are secondary structures of *architectural, mechanical* and *electrical components*, mounted on primary structures (buildings), with seismic weight less than 25% of effective seismic weight of the structure [1]. Past earthquake experience demonstrated participation of *NSEs* in the lateral load resistance and even alteration in the load paths of primary structures. Particularly, the architectural NSEs triples the lateral stiffness of building and reduce inter-storey drift by more than 2 times. [2]. Failure of acceleration sensitive NSEs occurs frequently after an earthquake because the dynamic features of the supporting structure are disregarded while calculating the seismic demand on NSEs [3]. Thus, modelling approaches of acceleration sensitive NSEs (Figure 1) are gaining prominence, to minimize *NSE* damages. There are mainly two modelling approaches of acceleration sensitive NSEs in buildings, namely (a) coupled or complete model and (b) decoupled model [4].

Figure 1 : Examples for acceleration sensitive NSEs [5]

While in coupled models, NSEs are modelled together with the primary structure, they are not in decoupled models. Despite permitted use of decoupled model in the absence of sufficient data during preliminary structural analysis of buildings, it leads to significant error especially in seismic demands on vertical lateral load resisting elements [6]. One of the primary inputs to calculate the force on *NSEs* is *Acceleration Floor Response Spectra* (*AFRS*) of the buildings obtained from dynamic analysis of buildings [7]. Several numerical studies had been carried out to understand the behaviour of NSEs under seismic loads using coupled and uncoupled models [7, 8, 9, 10, 11, 12, 13, 14, 15]. In some studies, benefits of coupled model over decoupled model were investigated; amplification of the displacement response (especially at top floor) were observed in coupled model when the fundamental frequency of the primary structure and the secondary structure are equal [16]. On the other hand, experimental investigations on decoupled model reported underestimation of actual acceleration experience by the NSE [17]. Further, behaviour studies of light subsystems supported on nonlinear structures using floor spectra of decoupled model without considering building-NSE interaction, torsion in buildings, and inelastic actions in buildings were also carried out [18, 19]. Torsion in supporting structures, in particular, amplifies response of the NSE depending on its location [20]. And, presence of large openings in the façade leads to torsional effects in supporting structure [21]. Also, contribution of higher modes depends on the characteristics of ground motion and the associated flexural overstrength of each mode [22]. An expression was also proposed considering nonlinear variation of FAF for a decoupled model [23]

Thus, there is an urgent need to complement the ongoing research using coupled model to predict building-NSE interaction. This paper presents numerical studies in this direction; complete (–coupled) modelling of rigid NSEs ($T_S \leq 0.06$s) is adopted in commercial software PERFORM 3D, to capture the seismic behaviour of NSEs and its interaction with the supported study building [1, 24]. The design parameters considered in this study are: (a) floor amplification factor at base of NSE, (b) component amplification factor at top of NSE, and (c) lateral force demand on NSE.

2 DESIGN SEISMIC LATERAL FORCE ON NSE

Expressions to estimate design seismic lateral force enunciated in international codes are summarized in Table 1. In general, *demand* and *performance* factors are used to determine the design seismic lateral force on *NSEs* [25]. When considered together, these factors constitute the *seismic coefficient* of NSE. While demand factor constitutes hazard factor, Component Amplification Factor (CAF) and Floor Amplification Factor (FAF), importance/risk factor and ductility Factor of the component are part of performance factor. Hazard factor represents the design acceleration of the site considered and CAF and FAF are empirical. Importance factor depends on the component functions, and ductility factor decides level of inelasticity allowed in the NSE under a seismic event. But, performance factors are mostly judgemental [25].

Codes	**Design horizontal lateral force**
ASCE 7:2016[1]	$F_p=(1+2z/h)0.4a_p I_p S_{DS} W_P /R_p$; $S_{DS} = 2/3\ F_a\ S_S$; $0.3 S_{DS} I_p W_p < F_p < 1.6 S_{DS} I_p W_p$
EAK 2000[26]	$H_p = (\varepsilon W_p\ \gamma_p)/q_p$; $\varepsilon = \alpha\beta\ (1+z/H)$; $\beta = 2/(1+(1-T_\pi /T)^2\ ) \geq 1$
EN 1998.1:2004[27]	$F_p = S_a W_a \gamma_a /q_a$; $S_a = \alpha\ S\ [3(1+z/H)/(1+(1-T_a/T_l)^2 - 0.5] \geq \alpha\ S$
NZS 1170.5:2004[28]	$F_{ph} = C_p\ (T_p) C_{ph} R_p W_p \leq 3.6\ W_p$; $C_p(T_p) = C(0)\ C_{Hi}\ C_i(T_p)$; $C(0) = C_h(0)\ Z\ R\ N\ (T,D)$
NZS 4219:2009[29]	$F_p = CW$; $C = 2.7\ C_H\ Z\ C_p\ R_c < 3.6$

Note : a_p component amplification factor; q_a behaviour factor of NSE; q_p reduction factor; z point of attachment of NSE above the base level; C lateral force coefficient; $C(0)$ site hazard coefficient for T = 0; $C_h(0)$ spectral shape factor; $C_i(T_p)$ component spectral shape factor at level i; C_p performance factor; C_{ph} part horizontal response factor; $C_p(T_p)$ horizontal design coefficient of the *NSE*; C_H, C_{Hi} floor amplification factor; F_a short-period site coefficient at 0.2s; H, h height of the building; I_p, γ_a, γ_p Importance factor of the NSE; $N(T,D)$ Near fault factor; R Return period factor; R_c component risk factor; R_p part risk factor/component response modification factor; S soil factor; S_a seismic coefficient; S_{DS} spectral acceleration at short periods; S_s spectral response acceleration parameter at short periods; T, T_l fundamental period of the building; T_a, T_p, T_π fundamental period of the *NSE*; W_a, W_p NSE weight; Z hazard /zone factor; α ratio of the design ground acceleration on to the acceleration of gravity; ε seismic factor

Table 1: Summary of code formulations

3 NUMERICAL STUDY

3.1 Details of the 3D RC building

The study Reinforced Concrete Special Moment Frame building modelled in commercial software PERFORM 3D is 7-storey tall with typical storey height 3.7m, 5 bays with bay length 5m in *X*—direction and 3 bays with bay length 3m in *Y*— direction, founded on medium stiff soil. Importance factor considered is 1.5, live load 3kN/m^2, steel and concrete grade Fe415 and M30, respectively. Buildings are designed to resist PGAs (Zone factors) 0.2g, 0.36g and 0.72g, following IS1893:2016 and IS13920:2016 provisions [30, 31]. Slab loads and brick masonry infill loads are distributed on beams, and rigid diaphragm action is enabled. Mander's confinement model is used for defining confinement effects of building elements [32]. Fibre modelling of inelasticity is used in select members in select locations of buildings.

3.2 Details of the NSE

In general, NSEs have weight less than 25% of effective seismic weight of supporting structure [1]. This study considers simple acceleration sensitive *NSEs* with fundamental period $T_S \leq$ 0.07seconds, modelled as a rigid cantilever with lumped mass at the top (stick model). NSE is located at the roof of the RC building (Figure 2), and is assumed to behave elastically. The equivalent height of NSE considered is 0.5 m. Parametric studies of NSEs with lumped mass of 1-10 percent total seismic weight of building (in increments of 1 percent) for each fundamental period of NSE (in increments of 0.01s) is carried out. Stiffness of the NSE is varied according to the required percentage of seismic weight of the building and desired fundamental period of the NSE.

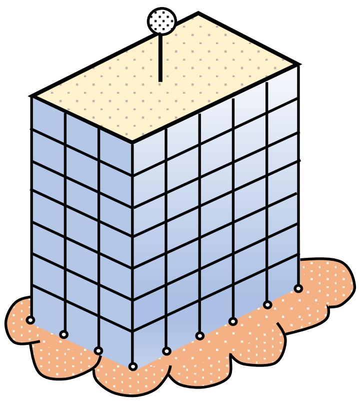

Figure 2: Schematic of *complete model* of NSE

3.3 Nonlinear Time History Analysis

Nonlinear time history analyses are performed of the complete model using 11 natural Ground Motions (GM) (Table 2) as per ASCE 07 and corresponding to 5% damped response spectrum presented in Figure 3 [1]. Spectral amplitude scaling is carried out at elastic natural period of the building designed for each design PGA [33]. A total of 2, 343 nonlinear time history analyses are performed to investigate the behaviour of NSEs and buildings under ground motions.

GM No.	*Event*	*Station*	*Year*	*M_w*	*PGA (g)*	*Epicentral distance (km)*
1	Kobe	Nishi-Akashi	1995	6.9	0.483	7.08
2	Loma Prieta	Hollister-South Pine	1989	6.9	0.370	28.80
3	Imperial Valley	Delta	1979	6.5	0.351	43.60
4	Chi Chi	TCU 047	1999	7.6	0.298	35.00
5	Hectormine	Hector	1999	7.1	0.265	11.66
6	Northridge	Downey- Co Maint Bldg	1994	6.7	0.229	47.60
7	San Fernando	Palmdale Fire Station	1971	6.6	0.133	25.40
8	Sikkim	Gangtok	2011	6.8	0.160	58.00
9	Uttarkashi	Bhatwari	1991	6.8	0.253	36.00
10	Fruili	Tolmezzo	1976	6.4	0.351	28.00
11	Peru	ICA	2007	7.9	0.278	140.00

Table 2: Characteristics of Ground Motions (GMs) considered in the study [34, 35, 36, 37, 38]

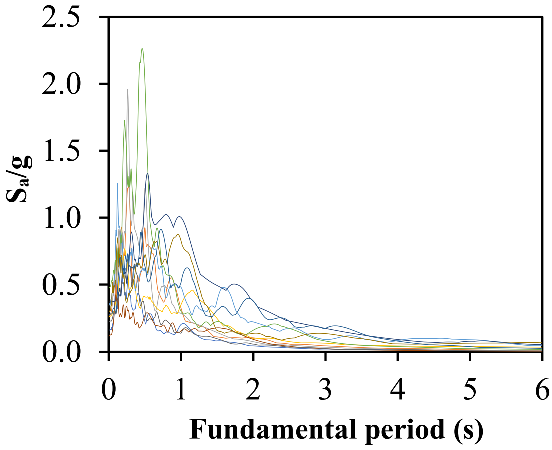

Figure 3: Elastic acceleration response spectra of ground motions considered in the study

4 DISCUSSION OF RESULTS

4.1 Floor Amplification Factor of the building

Floor Amplification Factor (FAF) is defined as the ratio of Peak Floor Acceleration (PFA) to the Peak Ground Acceleration (PGA). FAF represents the dynamic amplification of acceleration from ground to the floor location of interest where the NSE is attached, but without considering NSE. FAF is largely influenced by: (a) lateral load resisting system, (b) modal and damping properties of building, (c) structural configuration of building, (d) diaphragm rigidity, (e) stiffness, strength, and ductility of the building, and (f) the vertical location of NSE across the height of the building [39]. Variation of FAF across the floors of the designed buildings without NSE is presented (Figure 4). In Figure 4, for brevity, GM refers to ground motion whose numbers are represented in Table 2.

Investigation of results demonstrate less amplification of floor acceleration in buildings designed for 0.72g (stiff building) (Figure 4 (c)) compared to buildings designed for 0.20g (less stiff building) (Figure 4 (a)). It is observed that FAF (a) is nonlinear across the floors, (b) not a constant 3 for all the floors, and (c) is higher in lower floors. This behaviour is a departure from the provisions in design standards [ASCE 07:2016, EAK 2000, NZS 1170.52004, NZS 4219:2009]. Further, characteristics of earthquakes significantly alter the seismic behaviour of the building; of the 11 ground motions considered, amplification under Sikkim earthquake is more prominent (Figure 4). Amplification observed on the first floor exceeds the factor 3 of New Zealand code under Sikkim earthquake in building designed for 0.2g (Figure 4 (a)). Further, amplification in the third and fourth floor is more than on the second floor, and reduces towards top under most earthquakes. Furthermore, in all 3 buildings, at lower floors, FAF is significantly higher than the linear variation recommended by some international design codes.

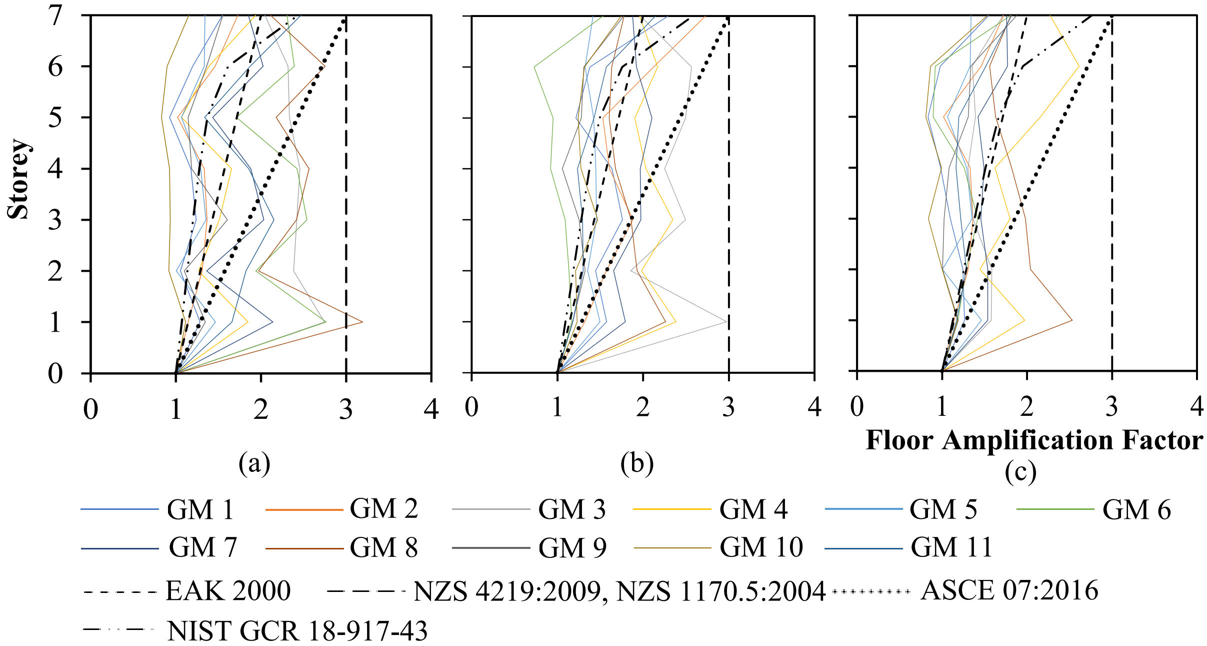

Figure 4: Variation of FAF in buildings designed for PGA: (a) 0.2g, (b) 0.36g, and (c) 0.72g

4.2 Roof response spectrum of the complete model

Roof response of complete model or system (both NSE and building modelled) represents the dynamic amplification of acceleration from ground to the roof where NSE is attached. Roof response spectrum of complete model, with NSE of fundamental period 0.04s and designed for 5% of seismic weight of building is presented in Figure 5. It is observed that spectral amplification increases at lower periods for all ground motions, and this increase is more prominent in buildings designed for higher PGAs. Maximum spectral amplification is about 8 for Sikkim earthquake at very low period (Figure 5 (c)); similar characteristics of Sikkim ground motion is observed in FAF also, thereby confirming the influence of ground motion characteristics on seismic behaviour of structural and non-structural elements. In particular, seismic resistance and behaviour of rigid NSEs with fundamental period less than 0.06 seconds are crucial and critically affected by ground motion characteristics.

4.3 Component Amplification Factor from the complete model

Component Amplification Factor (CAF) is defined as ratio of Peak Component Acceleration to Peak Floor Acceleration. CAF represents the dynamic amplification of acceleration from floor to the top of the NSE (Figure 6), and *average Component Amplification Factor* is average CAF under all considered ground motions. And, seismic weight ratio of NSE is the weight of NSE as percentage of seismic weight of the building.

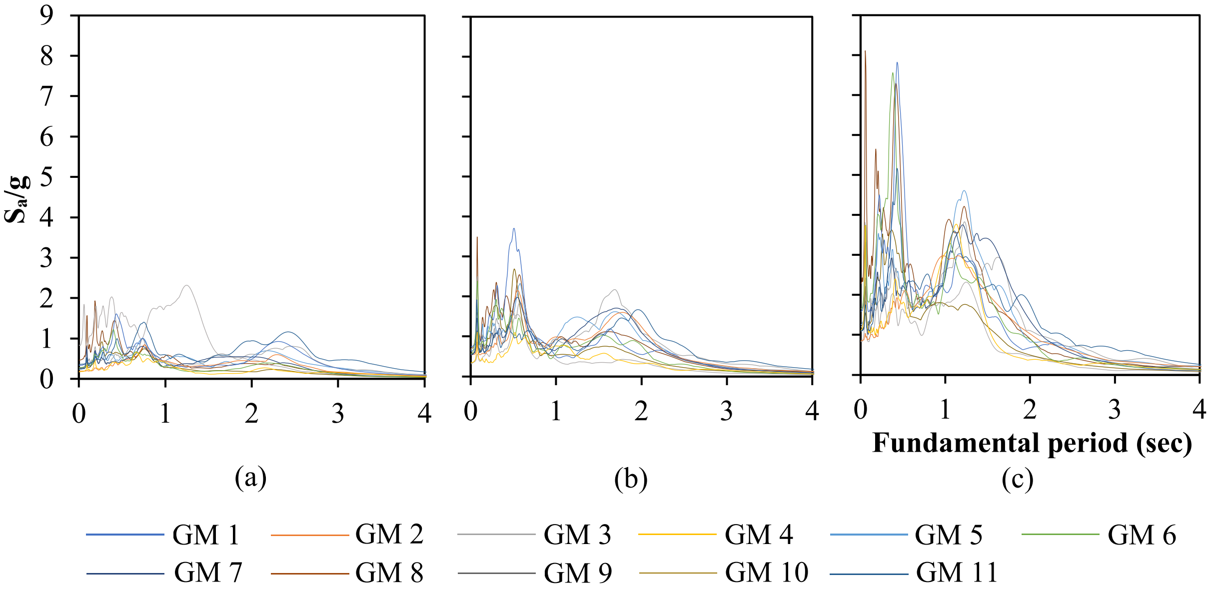

Figure 5: Roof response spectrum of system with buildings designed for PGA (a) 0.2g, (b) 0.36g, and (c) 0.72g

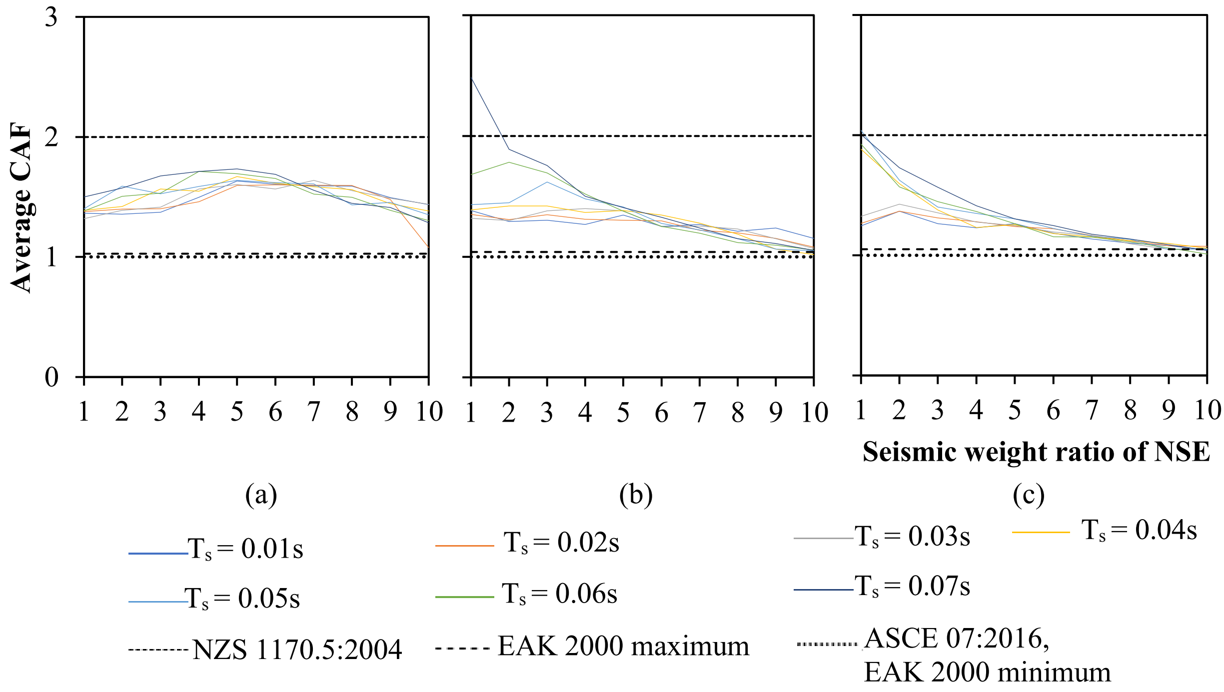

Figure 6: Variation of CAF in buildings designed for PGA (a) 0.2g, (b) 0.36g, and (c) 0.72g

It is observed that CAF increases marginally with increase in seismic weight ratio and reduces after 5% seismic weight ratio in building designed for 0.2g (Figure 6 (a)). Nevertheless, CAF is about 1.5 in all NSEs. And, CAF reduces significantly and approaches one rapidly in buildings designed for higher PGAs and more prominently in building designed for 0.72g (Figure 6 (c)). Also, NSEs with fundamental period 0.04s and above achieve CAF of about 2 for very stiff NSEs in building designed for 0.72g (Figure 6 (c)), and NSE with fundamental period 0.07s demonstrate a different behaviour with CAF above 2 in building designed for 0.36g

(Figure 6 (b)). This is complementing the ASCE 07 definition of rigid NSEs to be those with fundamental period less than 0.06s, because impending behaviour change is observed for the NSE considered above this range, in this study.

4.4 Design lateral force from complete model

Seismic coefficients for design of NSEs are first estimated using design code recommended equations (Table 3). And, the average lateral force demand on NSEs under 11 ground motions from the PERFORM 3D results, and seismic coefficient using the above average lateral force demand, are also estimated for comparison with Table 3 values (Figure 7). Eurocode and Greece code estimates minimum seismic coefficient for NSE with fundamental period 0.01s, and maximum value for NSE with fundamental period 0.06s (Table 3). For further comparison in this study, maximum values are used.

<table>
<tr><th>Codes</th><th>Parameter
Design PGA (g)</th><th>FAF</th><th>CAF</th><th>Component response factor</th><th>Importance Factor</th><th>Lateral Force demand (W_p)</th></tr>
<tr><td>ASCE 07:2016</td><td>0.20
0.36
0.72</td><td>3</td><td>1</td><td>1.5</td><td>1.5</td><td>0.26
0.44
0.71</td></tr>
<tr><td>EAK 2000</td><td>0.20
0.36
0.72</td><td>2</td><td>1.005-1.028
1.006-1.040
1.008-1.050</td><td>2.5</td><td>1.5</td><td>0.24-0.25
0.43-0.45
0.87-0.90</td></tr>
<tr><td>EC8 EN 1998.1:2004</td><td>0.20
0.36
0.72</td><td colspan="2">0.578
1.041
2.091</td><td>2.0</td><td>1.5</td><td>0.43-0.47
0.78-0.81
1.57-1.65</td></tr>
<tr><td>NZS 1170.5:2004</td><td>0.20
0.36
0.72</td><td>3</td><td>2</td><td>0.85</td><td>1.0</td><td>0.36
0.64
1.29</td></tr>
<tr><td>NZS 4219:2009</td><td>0.20
0.36
0.72</td><td>3</td><td>---</td><td>0.85</td><td>1.0</td><td>1.38
2.48
3.60</td></tr>
</table>

Table 3 : Lateral force estimate using international design standards

Due to comparable component response factor and the format of seismic coefficient estimate, ASCE 07 and EAK values are close and lesser than values estimated for other codes (Table 3). Further, it is observed that lateral force demand under actual ground motions decreases with increase in seismic weight ratio, but increases at a particular seismic weight ratio and increasing design PGA (Figure 7). This decrease with increase in seismic weight ratio is significant for NSEs with seismic weight ratio 5% and above. In addition, for 9% seismic weight ratio and above, lateral force demand is within NZS 1170.5:2004 in buildings designed for 0.72g (Figure 7 (c)), but exceeds in buildings designed for lower PGAs (Figure 7 (a) and (b)). On the other hand, NZS 4219:2009 values are not exceeded in any case. Similarly, seismic demand is lower than Eurocode estimate only in NSEs with 7% seismic weight and above in building designed for 0.2g, 6% seismic weight ratio and above in building designed for 0.36g, and 5% seismic weight ratio and above in building designed for 0.72g. And, in NSEs with fundamental period 0.04s and above, lateral force demand increases significantly in building designed for 0.72g (Figure 7 (c)).

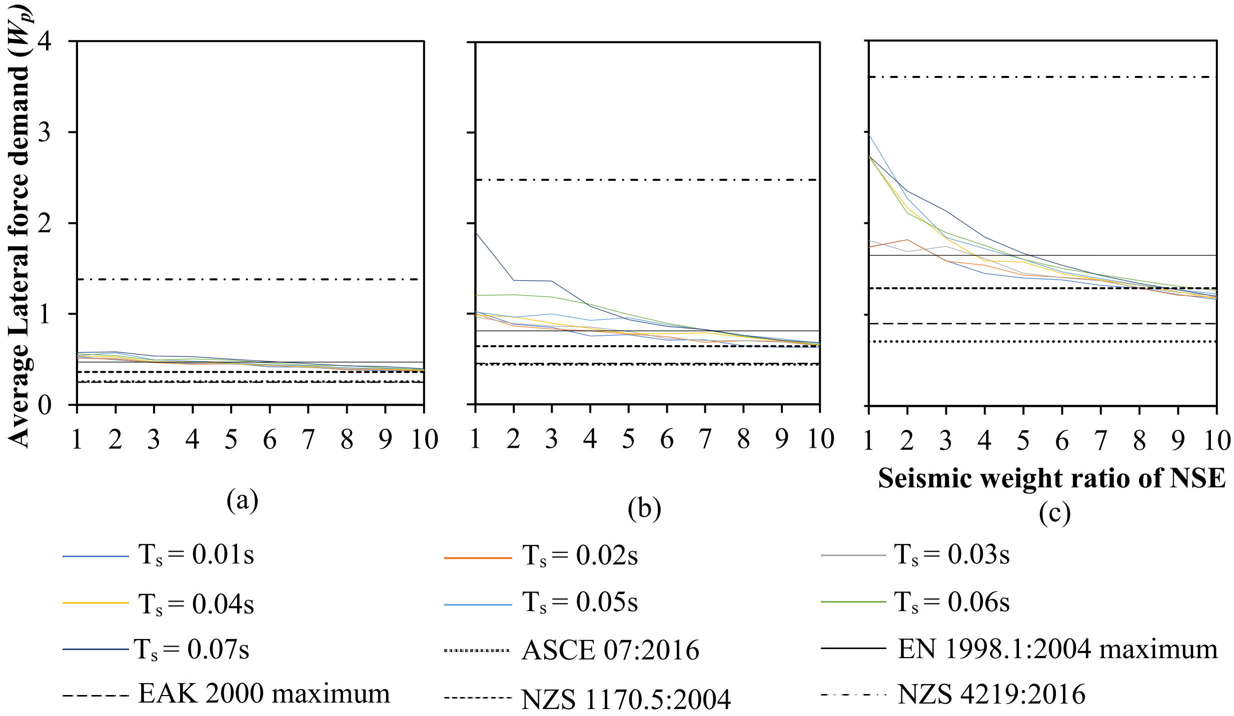

Figure 7: Variation of lateral force demand in buildings designed for PGA (a) 0.2g, (b) 0.36g, and (c) 0.72g

4.5 NSE – Building interaction

Providing NSEs with significant weight, stiffness and strength may affect the seismic behaviour of building supporting them or vice-versa. *Complete model* considering both building and NSE is expected to help understand the seismic behaviour interaction between the building and NSE. Linear and nonlinear behaviour of building- NSE interaction from the current work is briefly presented hereunder.

4.5.1. Linear elastic NSE – Building interaction

Variation of fundamental period of buildings designed for 0.2g, 0.36g and 0.72g are presented in Figure 8. In general, fundamental period of the buildings designed for 0.72g is lower than that of buildings designed for 0.20g. Further, increase in fundamental period of the building is proportional to the increase in seismic weight ratio of NSE, in all buildings. For *e.g.*, the fundamental period of building increased by 10% when seismic weight ratio of NSE was 10%, compared to the same building but without NSE. Further, this behavior is similar for NSEs with similar seismic weight ratio, but different stiffness. Thus, for NSEs with $T_s \leqslant 0.06$s, influence of NSE weight dominates its stiffness, in altering the fundamental linear behaviour of building. Thus, it is crucial to model NSE and building together in the linear elastic analysis to help predict floor demands (acceleration and displacement responses) realistically, for input to NSE.

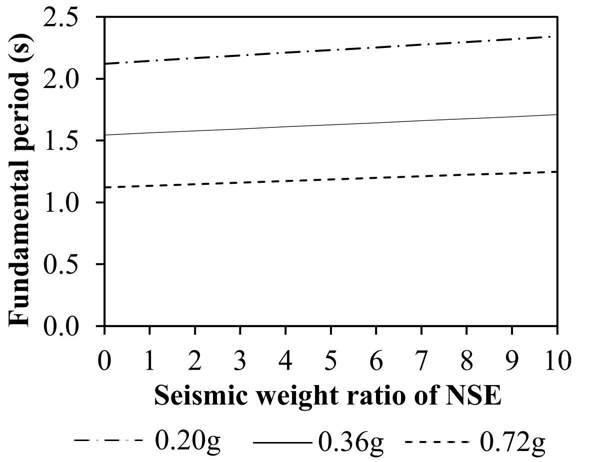

Figure 8: Variation of fundamental period of buildings

4.5.2. Nonlinear Building — Linear elastic NSE interaction

The average CAF decreases with increase in seismic weight ratio of NSE in buildings designed for 0.36g and 0.72g, but insignificant in the building designed for 0.2g (Figure 6). The is because building designed for 0.2g is less stiff compared to buildings designed for 0.36g and 0.72g (Figures 9 (a), (b) and (c)), thereby reducing rate of amplification of component acceleration. In particular, for Sikkim earthquake, PCA is about 6.8 (Figure 9 (c)). But, in the relatively stiffer building designed for 0.72g and 0.36g, an increase in the rate of amplification of component acceleration is observed especially upto 5% seismic weight ratio. Also, in general, it is observed that the amplification of component acceleration increases with increase in fundamental period of the NSE. Similar trend is observed in the lateral force demand in the above buildings (Figure 7). Thus, lateral stiffness of structure also influences the CAF and lateral force demand on NSE.

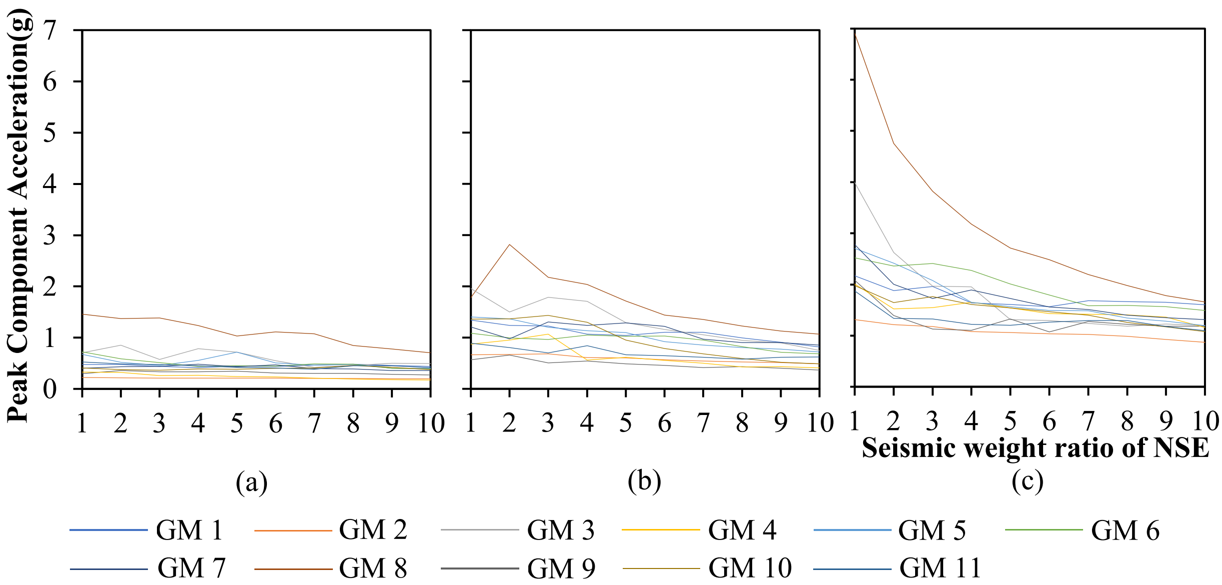

Figure 9: Variation of PCA in buildings designed for PGA (a) 0.2g, (b) 0.36g, and (c) 0.72g , T_s = 0.06s

5 SUMMARY AND CONCLUSIONS

In this study*, complete model* method of *NSEs* is adopted to capture the behaviour of *NSEs* more realistically. Numerical seismic behaviour of 3 sets of buildings designed for 3 values of PGAs and 7 sets of NSEs with varying seismic weight ratio are studied to draw inferences. Salient conclusions from the study are:

- FAF, CAF and lateral force demand depends on both characteristics of buildings, NSE and ground motions. In particular, fundamental period of the building and *NSE*, design PGA and lateral stiffness of building and floor response under ground motions influence the NSE demand parameters.
- FAF varies nonlinearly across height of building; it is higher in the lower storeys thereby increasing demands and damage vulnerability of NSEs. Thus, it is recommended that *NSEs* present in lower storeys of buildings should be designed for higher FAF compared to that used for upper storeys.
- CAF varies in the range 1-2, and hence the recommended CAF for rigid NSEs used in important buildings is 2.

However, more nonlinear dynamic numerical investigations on a spectrum of buildings and NSEs commonly used, supplemented by possible experimental validation, are required to complement the results obtained from this study.

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